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THE IMPACT OF FREEZING WINTER TEMPERATURES ON SEISMIC RESPONSE OF BRIDGE COLUMNS

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ABSTRACT

In a recent study, it has been shown experimentally and analytically that freezing temperatures can significantly influence the soil-foundation-structure interaction (SFSI) and structural response of bridge columns due to changes in material properties, especially those of soils. This paper investigates the seismic response of bridge columns subjected to different ambient conditions representing the summer temperature and several winter temperatures that varied the depth of the frozen soil. Using lessons learnt from previous research, analytical models were created to represent a two span prototype bridge system supported by a Cast-in-Drill-Hole (CIDH) foundation shaft, accounting for the effects of frozen temperatures on nonlinear column/pile response, damping, soil compressive non-linearity and gap development. In order to identify performance of the system at each temperature, models were subjected to seismic events with a range of intensities. As the ambient temperature deceased, the maximum bending moment demand increased and column/pile non-linearity was initiated at smaller seismic intensities. Compared to the 23°C case, the plastic region reduced by up 1.15 m as ambient temperature lowered, and the maximum pile shear increased as much as 73%. The maximum shear occurred in locations that were not critical during warm conditions, with increases of shear of up to 390%. This indicates brittle shear failure of the CIDH shaft if all possible temperature conditions are not taken into account.

Introduction

Previous experience has shown that large seismic events can occur during winter months in regions that experience seasonal ground freezing during these cold months but are otherwise unfrozen (e.g., New Madrid, Midwest of the USA, 1811-1812; Prince William Sound, Alaska, 1964; and Hokkaido, Japan, 2004). Only a small number of analytical and experimental studies have demonstrated the effect of seasonal freezing on soil-foundation-structure (SFS) response in regions that experience these conditions and are free of permafrost (Vaziri and Han 1991; Suleiman et al. 2006; Sritharan et al. 2007; Yang et al. 2007; Xiong and Yang 2008;

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Wotherspoon et al. 2009a, 2009b). Changes in the characteristics of the soil, concrete and steel reinforcement means that analysis of the SFS system using unfrozen properties will not be able to capture the response of structures during a winter earthquake. To be able to characterize the seismic behaviour of deep foundations that can experience seasonal freezing, both warm and cold temperatures should be given consideration in design.

This paper investigates the seismic response of bridge columns subjected to ambient conditions representing the summer temperature and several winter temperatures that varied the depth of the frozen soil. Using lessons learnt from previous research, analytical models were created to represent a two span prototype bridge system supported by a CIDH foundation. These models accounted for the effects of frozen temperatures on column/pile non-linearity, damping, non-linear cyclic behavior of soil and gap development. In order to identify the performance of the system at each temperature, models were subjected to seismic events with a range of intensities.

Background

The models in this paper were developed based on results obtained from an outdoor experimental program undertaken at Iowa State University to examine the response of large-scale bridge columns supported by CIDH shafts embedded in glacial till (Sritharan et al. 2007; Suleiman et al. 2006). Two identical test units were subjected to cyclic lateral loading during summer and winter months at average ambient temperatures of 23°C and -10°C, respectively. Data from testing showed that maximum moment, moment location, shear demand and length of plastic region were all significantly influenced by the temperature difference between the two test units.

Using measured material properties, an LPILE (Reese at al. 2000) model developed by Sritharan et al. (2007) was able to capture the envelope of the force displacement response, the maximum moment location, and the plastic hinge length of the test units during both test temperature conditions. Using data from other researchers, the monotonic response of the test units were modeled by these researchers for temperature conditions other than those present during physical testing. Wotherspoon et al. (2009b) were able to capture the cyclic responses of test units in warm and freezing conditions using Ruaumoko (Carr 2005) analytical models. Structural non-linearity, gap development adjacent to the foundation, and soil non-linear compressive characteristics were all incorporated into the models. Column force-displacement, gap opening at ground level, and the maximum moment and hinge length all matched well with the experimental results for both temperatures, indicating the accuracy of these models.

Bridge Model Characteristics

The bridge model used in this analysis uses much of its properties from the Iowa State bridge column/pile test units and represents a two-span structure supported by a single column/pile bent at midspan with no restraints at the abutments. The ground excitation is direction along the transverse axis of the bridge, meaning the bridge effectively acts as a single cantilever. The mass of the bridge superstructure was based on a 10% axial load ratio for the bridge column using unfrozen material properties. The bridge was assumed to be located in

Anchorage, Alaska, a seismically active area that experiences frozen soil to significant depths. This location is also outside the area where permafrost is encountered, meaning frozen soils are not present during the summer months.

The reinforced concrete column and foundation shafts had a diameter of 0.61 m, with an above-ground column length of 2.69 m and an embedded shaft length of 10.36 m. A longitudinal steel ratio of 2% was used along the entire column/foundation length. A 0.78% transverse reinforcement ratio was used in the column and in the shaft to a depth of 3.0 m, while below this the ratio reduced to 0.36%. Concrete and reinforcing steel properties are detailed later in the paper. The foundation soil was glacial till with properties defined using data from unconfined compression tests and CPT tests from the Iowa State experimental study (Sritharan et al. 2007).

The bridge model in Fig. 1 was developed using the Ruaumoko non-linear dynamic analysis program (Carr 2005) and is described in detail in Wotherspoon (2009) and Wotherspoon et al. (2009a). The Winkler spring concept was used to represent the soil surrounding the pile shaft. The column and foundation shaft in the Ruaumoko models were represented by 100 beam elements. Soil was modeled using non-linear spring and dashpot elements connected to the nodes at the end of the beam elements representing the foundation shaft. To represent the soil surrounding the pile, these elements were attached to each side of the foundation.

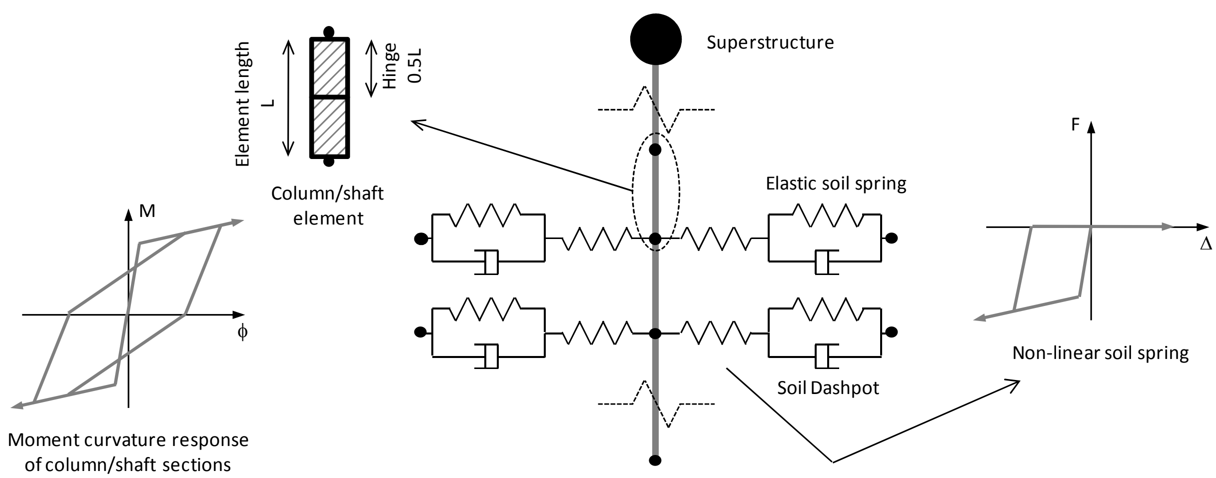

Figure 1. Details of the Ruaumoko bridge model

Column/Pile Modelling

Moment-curvature relationships of the column and foundation shaft sections were determined using fibre section analyses in OpenSees (McKenna et al. 2006). The effect of the superstructure axial load was accounted for in the fiber analysis assuming a 10% axial load ratio, which remained constant throughout the excitation. To represent the development and spread of plastic action in the column/foundation, the hinges at each end of the beam elements were made equal to half the element length. The moment-curvature properties of the column/foundation hinges were defined using the Modified Takeda hysteresis rule presented in Fig. 1 that approximates the section envelope response with a bi-linear curve (Otani 1981). Elastic structural viscous damping was modeled by defining appropriate Rayleigh damping coefficients to the column/foundation elements to provide 5% viscous damping.

Soil Modelling

Stiffness characteristics for the soil to a depth of 1.4 m were represented by p-y curves that were created from unconfined compression test data obtained for soil samples following the procedure presented by Reese & Welch (1975). Below this depth, p-y curves were created using the CPT data and procedures developed by Reese and Welch for stiff clay. Dashpot characteristics for the soil radiation damping in the seismic model were established based on elastic theoretical solutions reported for vibration of a pile by Gazetas and Dobry (1984).

At each foundation node, spring and dashpot elements were arranged in a series radiation damping configuration, with the soil separated into a plastic zone adjacent to the foundation and an elastic zone further from the foundation (Wang et al. 1998). The spring closest to the foundation, labeled non-linear soil spring in Fig. 1, models the gap opening and the non-linear compressive behavior of the soil. The compressive response was modeled using a bi-linear relationship fit to the p-y curves developed from test data. The elastic spring and the dashpot in Fig. 1 represent the elastic stiffness and radiation damping of the soil, respectively. Each soil spring was also prestressed to account for the horizontal insitu soil stresses present in the soil due to the soil overburden pressure (Wotherspoon 2009, Wotherspoon et al. 2009a).

Temperature Effects

To determine the influence of sub-zero temperatures on the seismic response of the bridge, models were developed for ambient temperatures of -1°C, -7°C, -10°C and -20°C. The response during non-frozen temperature conditions was represented using a temperature of 23°C. These temperatures were the same as those used by Sritharan et al. (2007), to determine the effect of frozen temperatures on the monotonic response of the Iowa State test units. Thermocouples installed in-ground during the experimental study measured frost depths of 0.46 m and 0.76 m for ambient temperatures of -7°C and -10°C. Frost depths of 0.076m at -1°C and 1.2 m at -20°C were assumed using the measured values and assuming surface frost at 0°C. Yang et al. (2005) measured a frozen soil depth of 1.5 m in Anchorage during the winter of 2005, indicating the depths analyzed were all within the range of possible conditions at the site. Data from testing showed that temperature within the frozen soil layer varied almost linearly with depth (Suleiman et al. 2006). Therefore, material properties were varied linearly through the frozen layer, resulting in unique properties at each depth. Frozen soil depths as a function of the cold temperature are summarized in Table 1.

Soil Characteristics

Unconfined compression tests on soil samples from the site at temperatures of -1°C, -7°C, -10°C and -20°C were used to create p-y curves using the approach of Reese and Welch (1975) and Crowther (1990). However, further modifications were made to account for the different crack development characteristics of the frozen soil identified during testing. Instead of radial cracks developing, the soil remained intact on the compression side and a pair of large tension cracks opened up adjacent to the pile shaft perpendicular to the lateral loading direction.

The p-y curve modification procedure to account for this tension crack development is detailed in Wotherspoon (2009) and Wotherspoon et al. (2009b).

Table 1. Summary of frozen soil depth and material properties for range of temperature conditions

Property	Temperature				
	23°C	-1°C	-7°C	-10°C	-20°C
Frozen soil depth (m)	0	0.076	0.46	0.76	1.2
Concrete compressive strength (MPa)	56.5	70.8	75.8	77.9	84.7
Steel yield strength (MPa)	471.5	481.9	485.1	487.4	492.4
Steel ultimate strength (MPa)	748.4	765.5	771.3	773.7	782.0

Column/Pile Characteristics

Concrete compressive strengths at 23°C and -1°C were measured using samples on the day of testing at Iowa State University. The remaining values were defined using the data presented by Lee et al. (1988). Steel reinforcement strengths at 23°C were determined using coupon testing, with properties in the sub-zero range estimated using the results from Filiatrault and Holleran (2001). A summary of concrete and reinforcement properties for the bridge column and shaft are included in Table 1. Along with the effect of temperature on material properties, the moment-curvature response of the frozen foundation sections accounted for the effect of frozen soil confinement. Using the methodology from Sritharan et al. (2007), additional confinement was provided to the section during fiber analysis to account for the effects of soil confinement on the moment-curvature response.

Seismic Analysis

The bridge model for each temperature condition was subjected to a 50% in 50 year, a 10% in 50 year and a 2% in 50 year earthquake appropriate for Anchorage, Alaska. For each event, seismic design spectra for this location were defined according to the Recommended LRFD Guidelines for the Seismic Design of Highway Bridges (AASTHO 2006) and USGS maps (Peterson et al. 2008, Wesson et al. 2007). The spectrum for the 50% in 50 years event was estimated using data from maps and the spectrum scaling methodology in FEMA 273 (1997). The site soil conditions corresponded to Site Class D – stiff soil in the LRFD Guidelines. Seismic design spectra for Anchorage are similar to that of Reno NV, and Salt Lake City, UT, and data from Bowles (1996) indicated frost depths of approximately 0.35 m and 0.6 m for these cities. As a result, some of these analyses also provide a good indication of the change in response at these locations between summer and winter months.

The 5% damped spectrum of each earthquake record was scaled to the appropriate design spectrum using the square root of the sum of the squares method detailed in FEMA 273 (1997). Fundamental period reduced as the ambient temperature reduced and the frozen soil depth increased, with values for 23°C, -7°C and -20°C equal to 0.934 secs, 0.803 secs and 0.740 secs, respectively. Kinematic interaction effects were ignored in this analysis as 1-D equivalent linear site analysis by Wotherspoon et al. (2009a) and Dutta et al. (2008) showed very little difference between transfer functions from bedrock to the ground surface for unfrozen soil and frozen soil to a depth of up to 3 m. Instead a constant seismic record was applied at each depth along the bridge foundation, which was unlikely to cause in considerable error as the emphasis of the study is placed on comparison of responses.

Bending moment

Fig. 2 presents envelopes of the maximum bending moment for each temperature condition for the range of seismic events. This provides a good indication of the change in response of each temperature condition with increasing seismic event. Variation in yield moment (M_y) with depth for the 23°C and -20°C cases are indicated in the figure. In general, results show that as the ambient temperature reduces, the maximum moment demand increases, the depth to maximum moment reduces, and the length of plastic region reduces. The 23°C case to the -1°C case has very similar characteristics, but once the temperature reduced to -7°C there was a shift in the response, with the remaining lower temperature conditions developing similar bending moment characteristics. During the 50% in 50 year event there was no non-linearity in the column and pile, while soil gapping and compressive yield developed for all temperature conditions. During the 10% in 50 year event the column/pile remained elastic for the 23°C and -1°C cases, with the rest of the temperature conditions resulting in column and pile non-linearity. As the seismic excitation increased to 2% in 50 years, the 23°C and -1°C maximum moment moved closer to the values in the other temperature conditions and the yield moment was reached. The -7°C, -10°C and -20°C cases only showed a slight increase in maximum moment values from the 10% in 50 year to 2% in 50 year event as yield moment had been reached in the smaller event.

Maximum moment for the 23°C and -1°C case during the 10% in 50 year event were equal to 776 kNm and 742 kNm, respectively. These values increased to 826 and 829 kNm during the 2% in 50 year event. As the temperature reduced further, the maximum moments increased, with values of 880, 899, and 921 kNm for the -7°C, -10°C and -20°C cases during the 10% in 50 year event. As these temperature conditions experienced column/pile non-linearity during this event, the increase in maximum moment during the 2% in 50 year event was only approximately 2%. Focusing on the 10% in 50 year event, the maximum bending moment depth for the 23°C case was 1.1 m below the ground surface, while the -1°C case was at 0.85 m below ground. As the temperature lowered, the maximum moment shifted much closer to the ground surface, occurring at a depth of 0.19 m for -7°C, -10°C and -20°C. During the 2% in 50 year event, the maximum moments remained at similar locations.

During the 10% in 50 year event, similar plastic hinge regions developed for the -7°C, -10°C and -20°C cases. For each of these temperatures, the plastic hinge region extended from approximately 0.10 m above the ground surface to 0.45 m below ground, providing a total length of 0.55 m. As indicated previously, no plastic hinge developed in the 23°C and -1°C case during the 10% in 50 year event. Comparison of the 2% in 50 year event provides a good indication of the significant effect of temperature on the plastic region. Similar plastic lengths were formed during the 10% in 50 year and 2% in 50 year event for the -7°C to -20°C cases. For the 23°C case, the plastic hinge region extended from 0.33 m to 2.03 m below the ground surface, a total length of 1.70 m, which is approximately 200% longer than the plastic region of the three lowest temperature conditions. The plastic hinge region for the -1°C case was 1.58 m in length, extending from 0.06 m to 1.64 m below the ground surface.

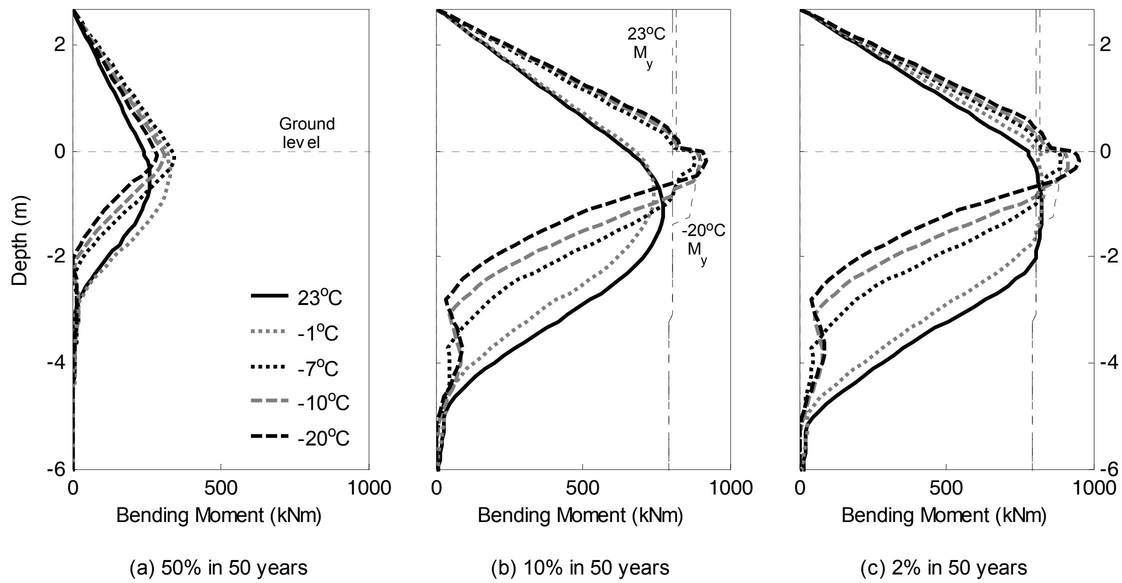

Figure 2. Maximum bending moment envelopes for each seismic event

Shear

The changes in the envelopes of maximum shear with temperature for each return period event are shown in Fig. 3. The increase in peak bending moment and reduction in plastic region as temperature decreases results in the larger peak shear shown in this figure. Data indicates that as the temperature decreases, the position of the maximum shear shifts upwards towards the ground surface, shifting the region of critical shear demand away from that expected during warm conditions. The 23°C and -1°C cases showed similar shear characteristics, but as the temperature decreased further there was a more gradual shift in the location and extent of the maximum shear force compared to the grouping of maximum bending moment characteristics shown previously.

Only very small increases in the maximum shear in the column and foundation with temperature are evident for the 50% in 50 year event. For the 10% in 50 year event, the maximum shear in the column for the -7°C case increased by 34% compared to the 23°C case, while in the foundation this increase was only 2%. At a temperature of -10°C, these values were equal to an increase of 41% and 24% of the 23°C values. Once the temperature had reduced to -20°C, there was a significant increase in the maximum shear in the pile, reaching a value 73% larger than the 23°C case. The reduction in temperature had a larger effect on the maximum shear in the pile than the column. For temperatures at -7°C and below, there were only small increases in shear demand between the 10% in 50 year and the 2% in 50 year event. Shear in the column for the 23°C and -1°C cases moved closer to the values developed in the lower temperature conditions.

For the 50% in 50 year event the maximum shear moved up from approximately 2 m below ground at 23°C to 0.7 m for -20°C. During the larger seismic events, the shifts in the position of maximum shear due to temperature effects became clearer, with this position shifting closer to the ground surface as the temperature reduced. During the 10% in 50 year event, the maximum shear position in the pile shifts upwards from 3.5 m for the 23°C case to 1.9 m, 1.1 m, and 0.7 m for the -7°C, -10°C and -20°C cases, respectively. These shifts are significant as they move the critical shear location to a position where the shear demand for the 23°C is not critical. At the maximum shear depth in the pile for -7°C, -10°C and -20°C for the 10% in 50 year event, shear demand was 113%, 360% and 390% larger than the shear in the 23°C pile at the same position. If such effects of seasonal freezing are ignored in design, then this area of the foundation will experience brittle failure in winter earthquakes with similar or larger intensities.

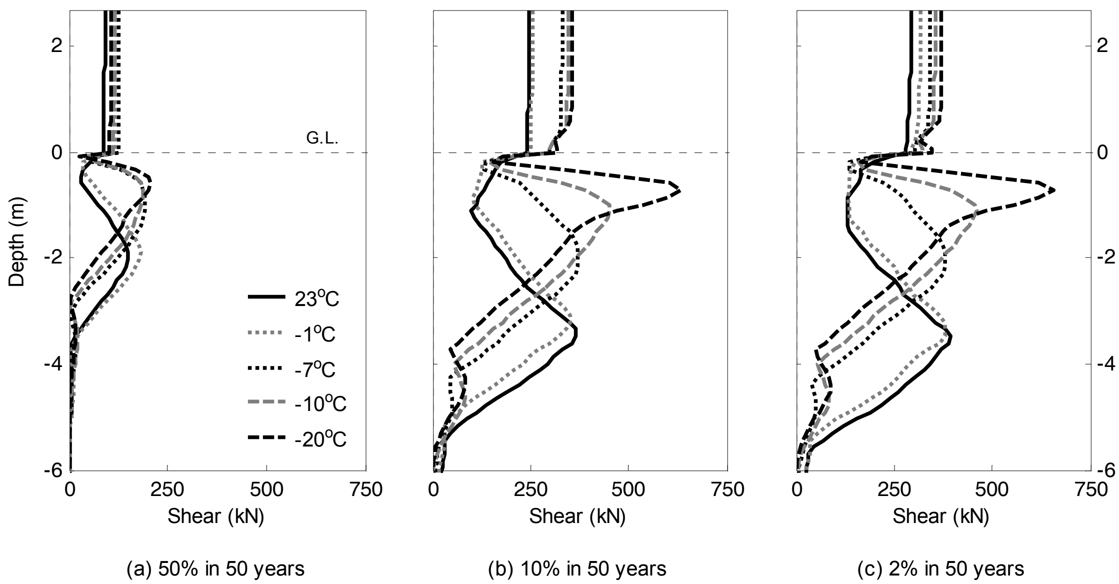

Figure 3. Maximum shear envelopes for each seismic event

Conclusions

Using analytical models from previous research, the effect of a range of ambient temperature conditions on the seismic response of a two-span bridge was investigated. Models accounted for the effect of frozen temperatures on the column/pile non-linearity, damping, soil compressive non-linearity and gap development. Analyses conducted over a range of event intensities highlighted the considerable impact of seasonal freezing on seismic response of the bridge and the importance of considering all effects of possible temperature conditions on design. Data from the 10% in 50 year and 2% in 50 year event showed that as temperature reduced:

- Maximum moment demand increased by up to 19%;
- Length of plastic hinge region reduced by as much as 1.15 m;
- Depth to maximum bending moment reduced as much as 0.91 m;

- Depth to maximum moment reduced up to 0.91 m;
- Maximum column shear demand increased up to 45%; and
- Maximum foundation shear force increased up to 73%; and
- Depth to maximum shear reduced as much as 2.8 m.

As the ambient temperature decreased, column/pile non-linearity was initiated at smaller seismic intensities. The maximum shear force occurred in locations that would not be critical during warm conditions, which could result in shear failure if all possible temperature conditions were not taken into account. For the -7°C, -10°C, and -20°C cases, this maximum shear was 113%, 360% and 390% larger than the 23°C value at the same depth.

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